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CONTROL OF EMBANKMENT MATERIAL BY LABORATORY TESTING

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SOIL MECHANICS AND FOUNDATIONS DIVISION

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PAPERS

CONTROL OF EMBANKMENT MATERIAL BY LABORATORY TESTING

By F. C. Walker[1] and W. G. Holtz,[2] Members, ASCE

Synopsis

The United States Bureau of Reclamation (USBR) laboratory has established procedures for standardizing the testing of material to be used in embankments of earth dam structures. This paper describes these tests and their applications, and their correlation with field data.

Compaction testing has been developed from the procedures established by R. R. Proctor, M. ASCE. The test has been slightly modified, and laboratory methods of determining the compaction effect of sheepsfoot rollers are described. The effect of rock present in the soil mixture is analyzed, and compaction testing of pervious materials is also considered. The laboratory tests led to the development of efficient compaction equipment and procedures for field use.

It is difficult to correlate data from permeability tests made in the laboratory with results of field tests. The conditions of these tests are outlined, and their merit is analyzed. These tests have resulted in the development of a theory of pore pressure that is valuable in appraising the characteristics of a given embankment material. The pore pressure principle is discussed, and the basic equations used in the determination of pore pressure are listed.

Consolidation and settlement tests can be carried out in the laboratory, and by using adjustments of the basic tests, it is possible to secure good correlation with field data.

Development of the triaxial shear test has produced a valuable tool in the analysis of soil samples. The apparatus and procedure of these tests is outlined, as well as modifications made necessary by pore pressure considerations.

Note.—Written comments are invited for publication; the last discussion should be submitted by June 1, 1952.

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The stability of an earth structure is dependent to a great extent on the moisture conditions at the time of placing. The theory of limiting moisture content is discussed, and it is related to the field control of moisture in embankments. The effect of pore pressure and rock content is analyzed, and several typical examples are cited.

Introduction

Within the field of soil mechanics there are several closely related but nevertheless distinct fields of endeavor, each of which has its own set of problems for which individual solutions have been developed. In the past few years, there has been a growing awareness that these individual solutions cannot be directly applied to related fields of endeavor even though they contain many useful ideas. In this field the USBR is primarily concerned with hydraulic structures such as large earth dams, and irrigation works, and to a lesser extent with nonhydraulic structures. As a result, the highest development of certain qualities of the soil that are largely incidental for other fields of endeavor has been found necessary. The observations reported in this paper and the solutions that have been developed should be considered as information that may aid in the solution of related problems but not as established procedures to be adopted without question.

In the development of designs for earth dams and irrigation canals, the primary emphasis of the USBR has been directed toward the retention of water. In the past it has been emphasized to the exclusion, or at least subordination, of other requirements. Actually, absolute imperviousness cannot be achieved and its relative value must be weighed against other equally important considerations. Therefore, any dam or canal embankment must not only have sufficient watertightness to serve the purpose for which it is intended, but also the structure must be stable within itself, and it must hold its form with time—that is, consolidation and settlement should remain within tolerable bounds for the life of the structure. Beyond these requirements are provisions for the utilization of modern construction equipment and procedures and the simplification of operation and maintenance.

What, then, are the characteristics of a soil that is suitable? The permeability of the soil under the condition of use must be known as well as its strength and how much it will change under load with the passing of time. The workability and durability of the soil in the structure are also important factors. There are laboratory tests available whereby values of permeability, stability, and consolidation may be determined for a given soil sample. There are no direct means available for defining workability or durability, but there are numerous index tests suitable for this purpose. Many investigators have been endeavoring to correlate tests such as grain size, density, plasticity index, penetration resistance, and others with performance. Certain advantages of such tests in indicating cheaply the relative properties of a soil are recognized, but no way has been discovered in which these values might be applied to the development of a rational design, except by purely empirical procedures.

This paper does not concern itself with the evaluation of these abstract characteristics. The laboratory procedures described herein are presented only in brief form, as they are described in detail elsewhere.[3]

All design theories require that a soil be homogeneous and isotropic. Those familiar with soils in practice know that there is considerable variation in the properties of the various samples tested for control purposes. The extent of these variations for different kinds of materials under normal construction control is under study, and the effect of these variations upon the performance of a given design is also of interest. In order to secure a valid comparison, the same standards of control have been maintained for all earth dams constructed since 1941. These dams, representing more than 30 separate fills, have been built of a great variety of materials under an extreme variety of construction conditions. These fills range from structures 40 ft high to some which are 450 ft high, and the volume of the structures range from 100,000 cu yd to 10,000,000 cu yd. The results of the numerous control tests have shown that a considerable spread must be tolerated, but it is possible to keep within pre-established limits, and, in some respects, the behavior of the completed structure can be predicted. There is also reason to believe that if it is possible to perform sufficient laboratory tests to establish reliable averages, these averages, properly interpreted, provide valid values for use in design analyses.

Compaction Testing

Adaptation of the Proctor Test.—When the USBR first established a laboratory for the study and control of soils to be used in earth embankments, the findings of Mr. R. R. Proctor[4] were used as a basis. Except for minor modifications, the tests used to establish compaction, penetration resistance, permeability, and consolidation performance standards are the basic tests still used. Proctor's original laboratory tests were performed on minus ¼-in. soil fractions (portions of the sample that pass a ¼-in. sieve). The soil was compacted into a mold in three layers, each layer being tamped by "25 firm 12-inch strokes using a rammer of 5½-pound weight with a striking area two inches in diameter." The mold used was "about 4 inches in diameter by 5 inches high" and had a 1/20-cu ft capacity. Mr. Proctor determined that a compaction cylinder of less than 1/20-cu ft volume gave density and needle data that could not be correlated properly with field compaction by sheepsfoot rolling. Compaction tests made in 1/30-cu ft molds, that have been widely used as a standard, generally have been found to provide slightly higher maximum density, slightly lower optimum moisture, and slightly higher penetration values than those found under field conditions because the material is more confined. In order to standardize the compaction test for USBR purposes, standards were adopted after considerable independent laboratory and field study. These standards include: (1) a 1/20-cu ft cylinder (about 4¼ in. inside diameter by 6 in. long); (2) a 5½-lb hammer, with 2-in. diameter tamping foot; (3) Hammer

[3] "Earth Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Appendix, Denver Federal Center, Denver, Colo.

[4] "Fundamental Principles of Soil Compaction," by R. R. Proctor, *Engineering News-Record*, Vol. 111, 1933, p. 245.

blow secured by 18-in. free drop; and (4) 25 blows on each of three 2-in. compacted layers.

The laboratory study established that the 18-in. free-fall drop of the hammer was approximately equal to the 12-in. firm stroke specified by Mr. Proctor. Penetration tests were made in the manner originally specified. An impact machine for making the standard compaction test by mechanical means has been fabricated, but the machine is used only for research purposes. It has been found that the manual test is satisfactory for all practical purposes, and the manual equipment can be readily handled in the field as well as in the laboratory.

Laboratory Analysis of Sheepsfoot Roller Compaction.—The standard laboratory impact procedure for establishing compaction characteristics had been questioned by some engineers who believed it did not give data comparable to that obtained on actual embankment using sheepsfoot rollers. Therefore, large-scale compaction tests using the sheepsfoot principle were undertaken in the laboratory in 1936. The equipment used is shown in Fig. 1. Essentially this machine consists of a hydraulic jack and an adjustable platform containing a pressure capsule for measuring the compacting forces. The adjustable platform allows the compaction of samples of various thicknesses. The compaction cylinder assembly consists of three parts—the base, the main cylinder, and a removable upper collar. The main cylinder is about 12 in. in diameter and 12 in. deep with a 0.75-cu ft capacity. The removable collar is 9 in. long. The specimens are placed in three 6-in. compacted layers, after which the collar and top layer were removed and the density determined on the bottom layers. The compaction test was made on several materials containing up to 10% rock particles ranging in size from $\frac{1}{4}$ to $1\frac{1}{2}$ in. Various moisture conditions were investigated as well as various conditions of coverage and knob pressures to develop moisture-density relations for the various compaction conditions. The results and implications of these tests will be discussed in detail in following paragraphs. However, as a matter of standards, it was found that the sheepsfoot compaction test gave data comparable to that obtained by the standard laboratory impact tests, and, therefore, it was felt that the standard (impact) test should be used for embankment materials, investigations, and embankment control.

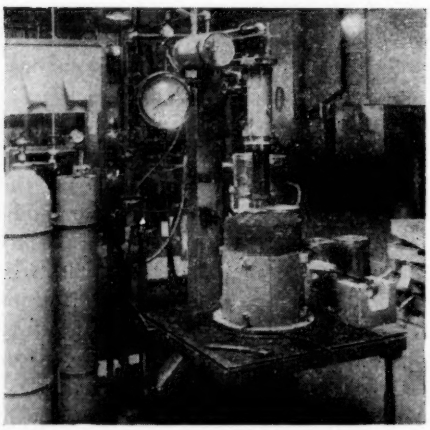


FIG. 1.—COMPACTION TEST APPARATUS

Theoretical Density of Rock and Soil Mixtures.—The standard laboratory compaction tests are conducted on the soil fraction of the total material.

This is assumed to be the portion of a sample that passes through No. 4 sieve. If the total material contains rock particles (particles larger than those passing the No. 4 sieve) the theoretical density of the total material can be computed from the formula:

$$D_t = \frac{1}{\frac{P}{D_r} + \frac{1 - P}{D_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which D_t is the dry density of soil and rock; D_s is the dry density of soil fraction as compacted; D_r is the dry density of rock fraction (specific gravity times 62.4); and P is the percentage of rock (expressed as a decimal). There are, however, physical limits to this method of calculation as shown by the example in Fig. 2. This figure shows graphically the effect on density of soil and rock fractions varying from $P = 0\%$ to $P = 100\%$ of the total material. The theoretical formula is valid only if the dry weight of rock per unit volume of the total material does not exceed the unit weight of the rock particles (generally taken as the dry rodded unit weight). If a material contains more than about 70% rock particles by weight of the total material, the relations as determined by the theoretical formula do not apply. Actually, this percentage varies from about 65% to 75%, depending upon the gradation, shape, and other like characteristics of the rock particles. The percentage of rock particles that a total material can contain without exceeding the physical limitations of the formula is called the theoretical upper limit of percentage of rock. This is shown by graphical presentation in Fig. 2. In this figure, the curve ABCD is the theoretical density curve (D_t) for total material (in pounds per cubic foot) as computed from the formula. The weight of rock (W_r) and weight of soil (W_s) in pounds per cubic foot of total compacted material, are shown as

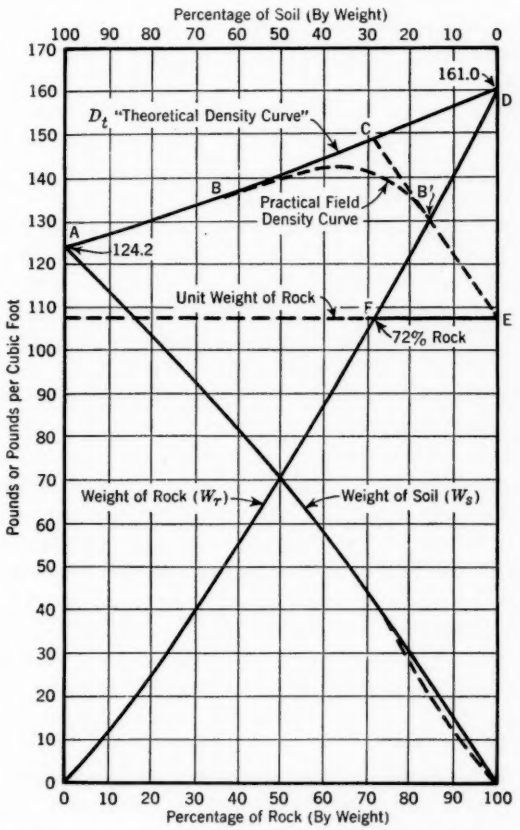

Percentage of Soil (By Weight)
100 90 80 70 60 50 40 30 20 10 0
Pounds or Pounds per Cubic Foot
170 160 150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0
161.0
D
D_t "Theoretical Density Curve"
C
B
B'
Practical Field Density Curve
A
124.2
Unit Weight of Rock
F
E
72% Rock
Weight of Rock (W_r)
Weight of Soil (W_s)
0 10 20 30 40 50 60 70 80 90 100
Percentage of Rock (By Weight)

FIG. 2.—LIMITATION OF THE THEORETICAL DENSITY FORMULA

continuous heavy curves which cross below the theoretical density curve. In this case the unit weight of the rock particles alone was found by laboratory experiments to be 108.0 lb per cu ft, and the maximum compacted dry density of the soil fraction (D_s) was 124.2 lb per cu ft (specific gravity = 2.68). Similarly, the density of the rock fraction was 161.0 lb per cu ft (specific gravity = 2.53). Since the unit weight of rock particles cannot exceed 108.0 lb per cu ft, the total material cannot contain more than 72% rock, if the rock voids are filled with compacted soil. The weight of rock (W_r) curve must then take the shape OFE. If this particular material contains more than 72% rock and if the gradation is maintained as the total material is compacted, the theoretical density will be reduced in accordance with curve CE and the curve ABCE becomes the limiting theoretical density curve for this material.

When soil and rock are compacted by field methods, or even laboratory methods, densities start to fall below the theoretical laboratory density when the total material contains about 30% rock, the percentage causing interference may be as low as 25% in some materials. Averages from several projects show considerable variation; in some cases theoretical densities were maintained for rock content as high as 50%, in extremely well-graded material. Individual measurements show an increased scattering in the zone beyond 40% rock content. Very few tests have been made on materials with rock content as high as 70%, but indications are that at this point 85% to 90% of theoretical laboratory density can be achieved. The average of the densities actually obtained would follow a curve similar to line ABB'E in Fig. 2.

Compaction Tests on Pervious Material.—It was recognized that the standard compaction tests would not apply to the coarse-grained noncohesive pervious materials. This situation did not concern the USBR greatly because the use of these materials was restricted to the outer layers of an embankment. Watertightness was not required under these conditions, satisfactory stability was presumed to exist under whatever method of compaction was used, and consolidation of any reasonable amount could be tolerated in the thin zones generally used.

The circumstances surrounding a number of dam sites chosen for development since World War II required either the use of very large zones of pervious materials or else the construction of extensive drainage blankets beneath the downstream section of the dam. Under such circumstances it was felt that the adequacy of compaction could no longer be left to chance. Accordingly, test procedures have been developed for the determination of relative density. Experiences with these procedures are still too limited to merit drawing any conclusions.

Standard laboratory tests have been tentatively adopted to determine the loosest and most dense states of pervious materials. The loosest density is determined by placing the dry granular soil in a prescribed manner with a hand scoop in a 0.1, 0.5, or 1.0-cu ft mold (depending on the maximum size of the particles) until the mold is filled to overflowing. The poured material is struck off with a straight edge, and the weight and density of the soil is determined. The highest density of the soil is determined for sands by compacting the granular soil in 1-in. layers, at optimum moisture, into the standard 1/20-

cu ft compaction cylinder using a compaction hammer having a 3½-in. diameter foot and a 10-lb drop weight. Fifty well-distributed blows of the drop hammer, falling 18 in. are applied to each layer. Vibration is used if this procedure produces densities equal to or greater than those obtained by the impact method or if gravel sizes are present; 0.5 or 1.0 cu ft cylinders are used for testing these coarser materials. The relative density (D_R), expressed as a percentage, is computed by the following equation:

$$D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which e is the in-place void ratio; e_{max} is the void ratio at loosest state; and e_{min} is the void ratio at most compact state. For convenient field use, the foregoing equation may be written:

$$D_R = \frac{D_{max}\,(D - D_{min})}{D\,(D_{max} - D_{min})} \times 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which D is the in-place density; D_{max} is the density in most compact state; and D_{min} is the density in loosest state.

The field control for these pervious materials is specified to obtain at least 70% relative density. Although laboratory studies have been conducted, tests and field observations are still in progress to further confirm this requirement.

Design of Field Compaction Equipment.—In an attempt to secure the compaction standards first outlined by Mr. Proctor[4] and subsequently modified for USBR practice,[5] continuously larger and heavier rollers were specified. Test sections of various types were tried in an attempt to arrive at a comparison between the standard laboratory compaction and the roller performance in the field with indifferent success. It was always possible to achieve a fair degree of comparison, but many erratic results were obtained, that have variously been ascribed to lack of sufficient construction control, excessive differences in the conditions between laboratory and field compaction, confinement effects, and influence of oversize materials.

USBR engineers finally designed a roller far heavier than anything then available. This roller was so heavy that only the largest tractors could pull it successfully. With such rollers it was possible for the first time to secure consistently compaction equivalent to the laboratory standard. In fact, the first reports were so encouraging that serious consideration was given to increasing this standard. Fortunately, no changes were made, so the information being gathered is still referred to the same standard.[5] Wartime interruptions with the resulting changes in construction practices and economics, and the loss of personnel with the prerequisite technical skill have all contributed to a reduction in the quality of compaction control that was being achieved in the first postwar years. Therefore, the safety margin of compacting ability

[5] "Earth Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Chapter VII, Denver Federal Center, Denver, Colo.

available in this roller has been extremely valuable. The result is that the type of compaction sought since 1936 is finally being successfully achieved but only with specific types of soil and only if reasonably good construction practice is followed.

The laboratory compaction tests with sheepsfoot type tampers, described previously, provide some interesting information on the comparison of laboratory and field data. The standard USBR roller provides knob pressures up to 490 lb per sq in. when fully ballasted with sand and water, as based on a total roller weight of 41,500 lb acting on 5% of 240 knobs, each with a 7.07-sq in. area. With the basic 12-roller passes usually specified, this roller provides a theoretical coverage of 70%, when the knobs are not worn, or 62% when the knobs are worn the maximum allowable amount.

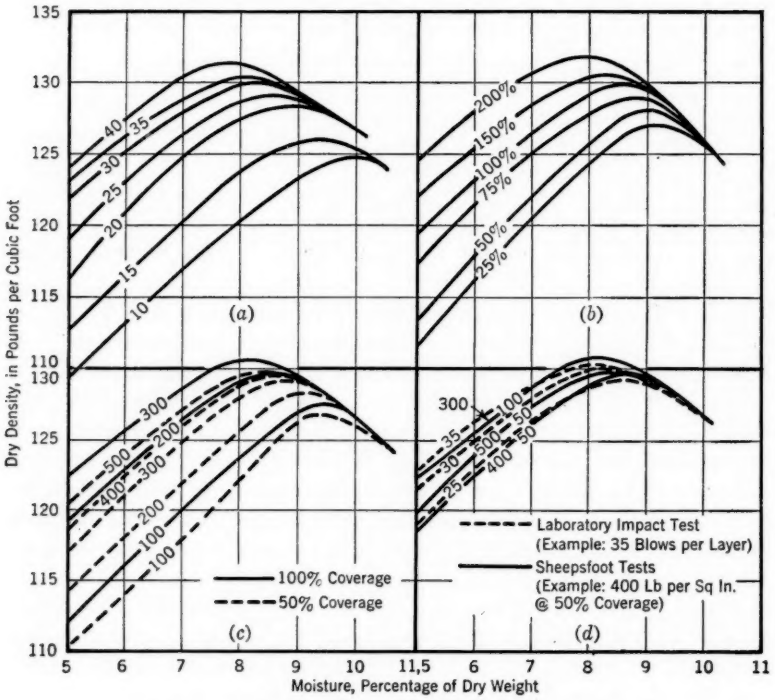


FIG. 3.—COMPARISON OF RESULTS OF COMPACTION STUDIES

Laboratory Standards for Compaction Tests.—By comparing the curves in Fig. 3, based on Platte River embankment material studies, it can be seen that the compaction characteristics for the sheepsfoot and impact compaction tests are quite similar. With 200 lb per sq in. knob pressures, a sheepsfoot roller coverage of about 80% is required to equal the results obtained by the laboratory standard impact test. Likewise, at 50% coverage, knob pressures of 350 to 400 lb per sq in. are required to equal the results obtained by the

laboratory standard test. On the basis of this information, about 29 blows per layer in the laboratory compaction cylinder are required to equal the sheepsfoot results under maximum knob pressures and 50% coverage or 35 to 40 blows per layer are required under maximum knob pressures and 70% coverage. Recognizing some loss of efficiency in field compaction, it has been estimated that laboratory compaction of 30 to 35 blows per layer gives compaction values similar to field roller compaction under maximum knob pressures and 70% coverage. For this reason, 33 blows per layer laboratory impact compaction is often used as a control for moisture limit tests and other detailed laboratory tests. It should be noted, however, that the difference in density and moisture relations obtained by the 25 blows per layer commonly used in laboratory compaction tests and in field control or the 33 blows per layer laboratory standard is relatively small, and the former test is quite satisfactory for design and control work.

There is no one best way to compact all kinds of materials. Instead, the USBR has tried to determine the need for compaction of any soil in accordance with the purpose it is to serve. Some granular coarse-grained soils could be compacted as well, or better, simply by the passage of a large tractor as they were with tractor and roller. Decreasing densities must be accepted with increased percentages of coarse material in otherwise well-graded mixtures. No evidence has been found that so-called over-compaction, first referred to by Theodore T. Knappen,[6] is a matter of concern. The expansive properties of bentonitic clays are recognized, but this type of material is normally avoided in the construction of earth dam embankments. Slight expansion of samples compacted in the laboratory have been noted when the load was removed after the samples had been loaded and saturated. There has been some evidence that the upper layers of a highly compacted fill demonstrate a slight rebound, but it is believed that the benefits of high compaction far exceed the disadvantages that such behavior may produce. There are, in fact, some who infer that the kind of compaction required by the USBR is inadequate.[7]

With the increase in compactive effort in the field it was found advisable to decrease the moisture used for compaction as determined by laboratory tests. Although the original intention was to reduce the opportunity for development of high pore pressure, this change also brought the moisture content closer to the optimum for the roller, avoided the danger of having soft fills when slight excesses of moisture occurred, and provided a greater margin of acceptable moisture limits for satisfactory compaction. The difficulties experienced with fill construction in 1930, when the puddle fill philosophy still dominated the picture instead of the practices used for embankment construction today, emphasizes the need for dry firm fills obtainable only with heavy compactors and limited moisture.

If the idea is accepted that the various performance characteristics of a given soil can be defined within reasonable limits if the density and moisture

[6] "Calculation of the Stability of Earth Dams," by Theodore T. Knappen, *Transactions*, Second Congress on Large Dams, Vol. IV, 1936, p. 505.

[7] "Laboratory Soil Compaction Methods, Penetration Resistance Measurements, and the Indicated Saturated Penetration Resistance," by R. R. Proctor, *Proceedings*, Second International Conference on Soil Mechanics and Foundation Eng., Vol. V, 1948, p. 242.

content are established, then laboratory performance and field performance may reasonably be comparable since experience shows that within established limits these conditions can be duplicated.

Permeability Tests

Development of Standard Technique.—Laboratory permeability tests of embankment materials are performed on the minus No. 4 fraction of the soil, using specimens 8 in. in diameter by 3 in. deep. The soils are compacted in steel cylinders at optimum moisture and maximum laboratory density in three equal layers, following the procedures originally outlined by Mr. Proctor but somewhat modified by the USBR. If the material will undoubtedly show a moderate permeability rate, greater than 1.0 ft per yr the modification consists of using a loading standard equivalent to 20 ft of fill, assuming that this will give a conservative value of permeability for the equivalent section of the embankment. If, however, the material to be tested is one in which the permeability is very low, the practice of using a loading equivalent to 100 ft of fill has been adopted as giving a more reliable indication of the character of the soil. If, upon further consideration, it is found necessary to use such material for the imperious section of a major dam, the permeability and consolidation characteristics throughout the entire range of loading are then examined.

In 1940 it was found necessary to consider for use in an embankment a soil that consisted of a mixture of about 50% gravel and 50% clay. Previous experience of the USBR led to the belief that in such a case the characteristics of the fine-grained portion would control the character of the mixture. In about this range of gradation the USBR standards of compaction would be attained only with great difficulty, if at all. It was deemed desirable, therefore, to test the complete material instead of only the minus No. 4 size so a testing procedure was devised for such materials. When materials containing a considerable proportion (about 25%) of particles larger than No. 4 size (those retained on a No. 4 sieve) are tested, large steel permeability cylinders 20 in. in diameter by 9 in. deep are used. The maximum particle size for this test is limited to 3 in. In this test the material is compacted in three equal layers, at laboratory optimum moisture for the mixture at the density anticipated for the field compaction operation. Loads are applied to both large and small specimens through springs, and facilities are available for consolidation measurements.

Permeability tests on undisturbed fine-grained soils are made on cylindrical specimens, $3\frac{1}{4}$ in. in diameter by 3 in. to 9 in. long. These specimens are placed in rubber sleeves, and the sleeves are attached to perforated end plates. The specimen and container assembly is placed in a pressure chamber and surrounded by water. A pressure slightly in excess of the percolating water pressure is applied to the chamber to hold the sleeve tightly against the specimen. Relatively air-free or de-aired water is permeated through the specimen by gravity or pressure flow until a constant rate is obtained. In all cases the test conditions are selected to duplicate the actual field conditions as closely as possible.

Field Tests.—The idea has been advanced that flow through soils followed a certain pattern.[8] Laboratory tests on sand models and electrical analogies provided a means for determining the flow lines for a given design, and the La Place equation provided a means of evaluating the loss through the designed section. To prove the reliability of these interpretations, wells were sunk into some of our older dams to locate the position of the uppermost (or phreatic) flow line. Assuming that bedrock provided the lowermost flow line and also that the quantity of water passing the dam could be determined, it would then be possible to check the permeability of the materials in a dam.

It soon became evident that the water levels measured in these wells did not correspond to any preconceived ideas for the location of the line of saturation. It seemed that with the low permeabilities of the surrounding soil in the embankment, these wells acted either as reservoirs or drains and some other method would have to be devised that did not require a supply of water from the embankment. Subsequently, pressure cells that met this requirement were installed. Although it is now believed that the pressures revealed by these cells were reliable, when the cells remained operative, the information that would enable a check of laboratory permeability tests was not obtained. Permeability no longer seems important in connection with embankments since material having a permeability so low that loss through the embankment is economically negligible has always been available for the core of the dam. If, in actual construction, field permeabilities are a hundred times as great as in the laboratory, losses through the usual embankment would still be unimportant.

Foundation permeabilities have been even more difficult to correlate with laboratory measurements. This must be expected, for it is virtually impossible to reproduce either field conditions in the laboratory or exercise laboratory control in the field. If field and laboratory tests show results of the same order of magnitude, excellent correlation is considered to have been achieved. Nevertheless, many surprisingly accurate estimates of leakage into foundation excavations have been made from rather questionable data combined with judgment and experience. Although attempts to check laboratory permeability in the field have been unsuccessful so far, they have been instrumental in disclosing a phenomenon of embankment behavior whose importance in large embankments far exceeds any permeability considerations. This is the development of the pore pressure concept.

The Pore Pressure Concept

Background of the theory.—The significance of pore pressure and its effect on the stability of a soil mass was recognized by F. F. Smith, J. H. A. Brahtz, and L. W. Hamilton of the USBR during the early part of 1936. It was the result of an effort to rationalize observations made during field permeability testing and to interpret laboratory shear tests. The concept of pore pressure and its effect on the stability of a soil mass were not new in soil mechanics at that time. There were numerous references to the phenomenon by other

[8] "Seepage Through Dams," by Arthur Casagrande, *Journal*, New England Water Works Assn., Vol. 51, 1937, p. 131.

authorities, including the writings of Karl Terzaghi,[9] Hon. M. ASCE. However, detailed descriptions of the phenomenon, as well as supporting data, were seldom given. To obtain the data necessary to support the concept, a testing program that eventually consisted of numerous shear and pore pressure tests was undertaken by Mr. Hamilton in cooperation with Messrs. Brahtz and Smith. The preliminary findings of this program were published in 1939.[10] Since that time considerable additional research work had been carried out, although this work was later curtailed to some extent during World War II and the post-war years because of personnel limitations and heavy construction work programs. Later research work of the USBR in this field has not been published, but the results of actual pore pressure measurements in earth dams constructed by the USBR were contained in an article prepared for the Second International Conference on Soil Mechanics and Foundation Engineering.[11]

Theoretical Derivation.—Pore pressure, as defined for the purposes of this paper, is the internal hydrostatic pressure or the pressure in the pore fluid of a soil mass. Pore pressure develops in a soil mass as consolidation from loading takes place. As the shearing strength of soil due to frictional resistance depends on the contact pressures between the soil grains, loss of strength takes place as pore pressure develops since pore pressure reduces the particle contact pressures. This simple relationship can be expressed as follows:

$$f = \bar{f} + u \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which f is the total stress (or fill load); $\bar{f}$ is the effective stress (grain to grain pressure causing consolidation); and u is the pore pressure. The loading of a soil mass produces a volume change that must be accompanied by either a corresponding extrusion or compression of the pore fluid. Thus, if the soil is of low permeability, compression of the pore fluid will result. A soil mass is made up of solid soil particles, water, and air, and the pore fluid is defined as the water and air components of the mass. Within the normal range of pressures encountered, both the soil particles and water are practically incompressible. Therefore, the volume change occurs as a compression of the air, and pore pressures are produced even though the voids are not filled with water. The magnitude of the pore pressures developed in a soil mass is therefore dependent upon: (*a*) the compressibility of the pore fluid; (*b*) the volume change of the mass under given loads; and (*c*) the permeability of the mass and the surrounding material.

The pore pressure resulting from the consolidation of a soil mass can be computed by the following equations

$$u_c = u_a \left(\frac{V_a + h\, V_w}{V'_a + h\, V_w} - 1 \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$

or

$$u_c = \frac{u_a\, \delta V}{V_a + h\, V_w - \delta V} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5b)$$

[9] "Principles of Soil Mechanics," by Charles Terzaghi, *Engineering News-Record*, Vol. 95, 1925, p. 1029.

[10] "The Effects of Internal Hydrostatic Pressure on the Shearing Strength of Soils," by L. W. Hamilton, *Proceedings*, American Society for Testing Materials, Vol. 39, 1939, p. 1100.

[11] "Ten Years of Pore Pressure Measurements," by W. W. Daehn and F. C. Walker, *Proceedings*, Second International Conference on Soil Mechanics and Foundation Eng., Vol. III, 1948, p. 245.

in which u_c is the pore pressure after consolidation (gage pressure); u_a is the initial pore pressure (usually considered as absolute atmospheric pressure); V_a is the initial volume of free air in the soil mass (percentage of initial volume); V'_a is the consolidated volume of free air in the soil mass (percentage of initial volume); $\delta V = V_a - V'_a$ which is the volume change caused by consolidation (percentage of initial volume); V_w is the volume of water in soil mass (percentage of total initial volume); and h is the capacity of water to dissolve air as derived from Henry's law (approximately 0.02).

Eq. 5*a*[11] is similar to Eq. 5*b*, but is expressed in a slightly different manner more convenient to use.[12] Both equations are based on Boyle's law for compressibility of gasses and Henry's law for the solubility of air in water at constant temperature. In using these equations for computing pore pressures, the assumption is made that, if a soil containing air, water, and solids is loaded without permitting drainage, it will consolidate; and the consolidation occurs as a volume change in the air content. The air will not only develop pressure according to Boyle's law, but it will dissolve in water according to Henry's law by a weight proportional to the pressure and the volume of water present.

This method is used to estimate pore pressures in earth dam structures, and good correlations with actual field and laboratory measurements have been obtained.

Consolidation and Settlement Tests

In order to distinguish between consolidation and settlement, as applied to embankments and foundations, the change in volume of an embankment is referred to as consolidation, and the same process in a foundation is referred to as settlement.

In 1938 the USBR began the installation of telescoping pipe apparatus[5,12] in the major dams built under its jurisdiction. With this apparatus it is possible to measure the foundation settlement at the point where the apparatus is installed and also to measure the consolidation in any increment of the embankment bracketed by crossarms. It is also possible to correlate consolidation with time and load. Attempts have been made to measure fill consolidations to accuracies approaching 0.001 ft, but the measurements were found to be so erratic that no use could be made of this refinement. Therefore, measurements have been restored to the former standard of 0.01 ft. Individual measurements deviate quite widely from the average; but in cases in which a continuous record is established either with time or depth, it is possible to draw an average performance curve that may be expected to approximate the true conditions.

Laboratory consolidation tests on fine-grained soils are conducted in fixed ring consolidometers on specimens $4\frac{1}{4}$ in. in diameter by $1\frac{1}{4}$ in. high. For foundation soils specimens taken at the site are cut to fit the consolidometer rings, and for tests on embankment soils the materials to be tested are compacted into the container at anticipated field moisture and density conditions. The loadings, conditions of saturation, and other factors used during the test are dependent on the problem to be studied and on the anticipated field conditions. When embankment materials containing more than 25% rock

[12] "Estimating Construction Pore Pressures in Rolled Earth Dams," by J. W. Hilf, *Proceedings*, Second International Conference on Soil Mechanics and Foundation Eng., Vol. III, 1948, p. 234.

particles are to be tested, the 20-in. diameter percolation-settlement cylinders are used.

Some of the first attempts to correlate field and laboratory data showed an extremely wide divergence. When laboratory tests of an embankment material showed potential consolidation in the neighborhood of 9%, field measurements showed less than 2% consolidation. It was evident that some adjustment was required. The laboratory tests were performed on samples limited to a minus No. 4 size, whereas the material used in the fill contained material up to 3 in. in size and the plus No. 4 material comprised 30% to 70% of the total. Assuming that the coarse-grained fraction did not contribute to the measured consolidation, it was possible to reduce the discrepancy between field and laboratory observations by about one half. It was also noted that quite high pore pressures were being measured in this embankment. These pressures would act to reduce the effective consolidating load and would not be present in the small laboratory samples as drainage was provided. By making this adjustment it was found that close agreement between laboratory tests and field observations could be obtained on the basis of averages.[13]

Based on settlement and consolidation records obtained to date, there is reason to believe that the camber (that is, the slight longitudinal hump, placed on a dam during construction) will compensate for about 100 years of consolidation and settlement, assuming an exponential time-consolidation relationship. In a properly compacted dam, the determination of potential settlement from laboratory tests thus becomes relatively unimportant. On the other hand, as higher and higher dams are built, consideration of pore pressure as a force tending to destroy stability becomes very important. It is, therefore, assumed that the laboratory field relationship on consolidation is valid enabling the use of laboratory tests to determine the magnitude of pore pressure to be anticipated in the fill under various placement conditions.

Shear Tests

Testing Conditions.—All shear testing is carried out by the triaxial test method. Various size specimens are used, depending upon the type of samples submitted and the problem to be investigated. Specimens varying in size from $1\frac{3}{8}$ in. in diameter by $2\frac{3}{4}$ in. high to $3\frac{1}{2}$ in. in diameter by 9 in. high are generally used. The smallest specimens are used for foundation materials since several specimens can be cut from the same horizon of a 6-in. diameter core. These specimens are always used when testing impervious clays since the short specimen allows more rapid equalization of pore pressure throughout the specimen and thus provides a reasonable testing period. The larger specimens are generally used for testing remolded embankment materials. These specimens are more suitable for this type of work because it is difficult to mold a small specimen to a uniform density condition. The laboratory control specified for companion shear specimens of this type is 0.5 lb per cu ft density and 0.1% moisture content.

The triaxial shear tests are usually conducted on sealed specimens with pore pressure measurements being taken at the ends of the specimens throughout

the test. The shear values reported are based on effective pressures and represent the shear characteristics of the materials at zero pore pressure conditions. Preconsolidation prior to shear testing with or without drainage is performed if required by the data to be assembled. When tests are required on foundation specimens that are thoroughly saturated, unconfined compression tests are made to determine the approximate cohesion of the material. The shear tests described are conducted on the soil fraction of the material. The need for triaxial tests on materials containing rock particles is recognized, and equipment is being designed for conducting shear tests on materials containing up to 3-in. size particles. (Equipment for testing coarse-grained soils in specimens up to 9-in. diameter × 22-in. length is now in use.)

During the earlier stages of the USBR laboratory (1933 to 1936), direct shear tests were conducted to determine cohesion and friction values since it was recognized that the stress conditions induced by the direct shear test were not comparable to the stress conditions induced in an actual structure. Specimens that varied in size from 8 to 144 sq in. of shearing area were used. The larger specimens were used to test materials containing rock particles up to 1½ in. in size. It was noted in these particular tests that there was no change in the shear values of materials containing up to 10% rock particles over the shear for the soil fraction that contained no particles larger than No. 4 size.

Effect of Pore Pressure.—In an effort to interpret shear test data and find the explanation of the variations caused by different test procedures and moisture-density conditions, the pore-pressure concept was developed. This led to the principle of approaching the soil stability problem from the viewpoint of pore pressure. As research proceeded, it became evident that pore pressure could be neglected only when volume changes were so controlled that the pore fluid would drain from the specimen at the same rate as the volume change took place. When materials of low permeability were tested, it was found that the time required for testing was increased to such an extent as to make the test practically impossible.

One of the first indications of the effect of pore pressure was the fact that direct shear test results on soils that were not free-draining did not give a straight-line relation between the normal applied load and the corresponding shearing resistance. For the lower normal loads the consolidation of the mass was small; consequently, the compression of the pore fluid was so negligible that the pore pressure was relatively ineffective in reducing particle contact pressures. As the normal loads were increased and the consolidation of the mass became greater, the pore pressure increased and a resulting loss of effective particle contact pressure took place. Thus, the plot of applied normal load against shearing strength is a curve having its greatest slope at the lower loads and a lesser slope at the higher loads. This is also true when the moisture content of the mass is increased, the shearing strength becoming less as the water content is increased (or the air content decreased) because the pore fluid is less compressible and builds up to greater amounts under load.

Laboratory test specimens are subjected to stress under considerably different conditions than may prevail in the prototype when pore pressure is

present. For example, the laboratory specimen is tested in a matter of hours, whereas the loading process in the prototype may extend over several years. Drainage conditions are also entirely different. In such circumstances it would be futile to attempt to use the results of laboratory shear tests without pore pressure correction in the design of a large earth dam with any expectation of a similar behavior in the prototype.

It was necessary, therefore, to select a shear test and to perfect testing techniques that would provide the control desired. Methods and equipment of measuring pore pressure during the test had to be developed so the true shearing strength of the material could be determined. The triaxial shear test was considered excellent for this purpose because drainage could be controlled and pore pressure measurements taken.

The Triaxial Shear Test.—A great deal of effort was expended on the development of equipment and techniques for determining the true shear characteristics of a soil mass by the triaxial shear test method. From these developments it has been found that a straight-line relationship between effective normal load and the corresponding shear resistance generally exists up to the condition of complete saturation for remolded soils. This is true provided the line of limiting shear resistance is constructed on the basis of effective stresses and the stresses are determined at the actual time of failure of the specimen. The point of maximum principal stress ratio, which corresponds closely to the point of minimum volume, is used as the failure criterion.[13] Many investigators do not believe that clay soils exhibit true frictional resistance, but the results of many triaxial tests on numerous remolded soils by the USBR seems to indicate a general straight-line relationship, providing effective stresses and proper failure criterion are used. This principle is illustrated in Fig. 4. The numbers at the ends of the curves in Fig. 4 denote the applied lateral pressure, and the vertical lines indicate the point of maximum stress ratio. It is recognized that the straight-line relationship between normal load and shear resistance does not hold for all types of soils and conditions in the lower load range. For example, natural loess soils of low density do not develop normal shear characteristics until some normal load is applied that will break down the natural loose structure and bring the particles into contact. In this case, a definite break occurs in the line of limiting shear resistance at that loading, after which the line becomes on of normal constant slope.

A three-dimensional (consolidation-pore pressure-permeability) test was developed to study volume change against pore pressure of remolded and undisturbed soils under load. The test is also well adapted for determining the permeability of undisturbed soils as previously mentioned. In this test, cylindrical soil specimens in rubber sleeves are sealed to perforated metal end plates and are subjected to chamber pressures only, and the relation between volume change and pore pressure is studied under sealed and drained conditions. Excellent correlations between computed pore pressures and pore pressures measured during the test have been obtained.

[13] "The Use of the Maximum Principle Stress Ratio as the Failure Criterion in Evaluating Triaxial Shear Tests on Earth Materials," by W. G. Holtz, *Proceedings*, American Society for Testing Materials, Vol. 47, 1947, p. 1067.

Limiting Moisture Concept and Modified Control Based on Laboratory Studies

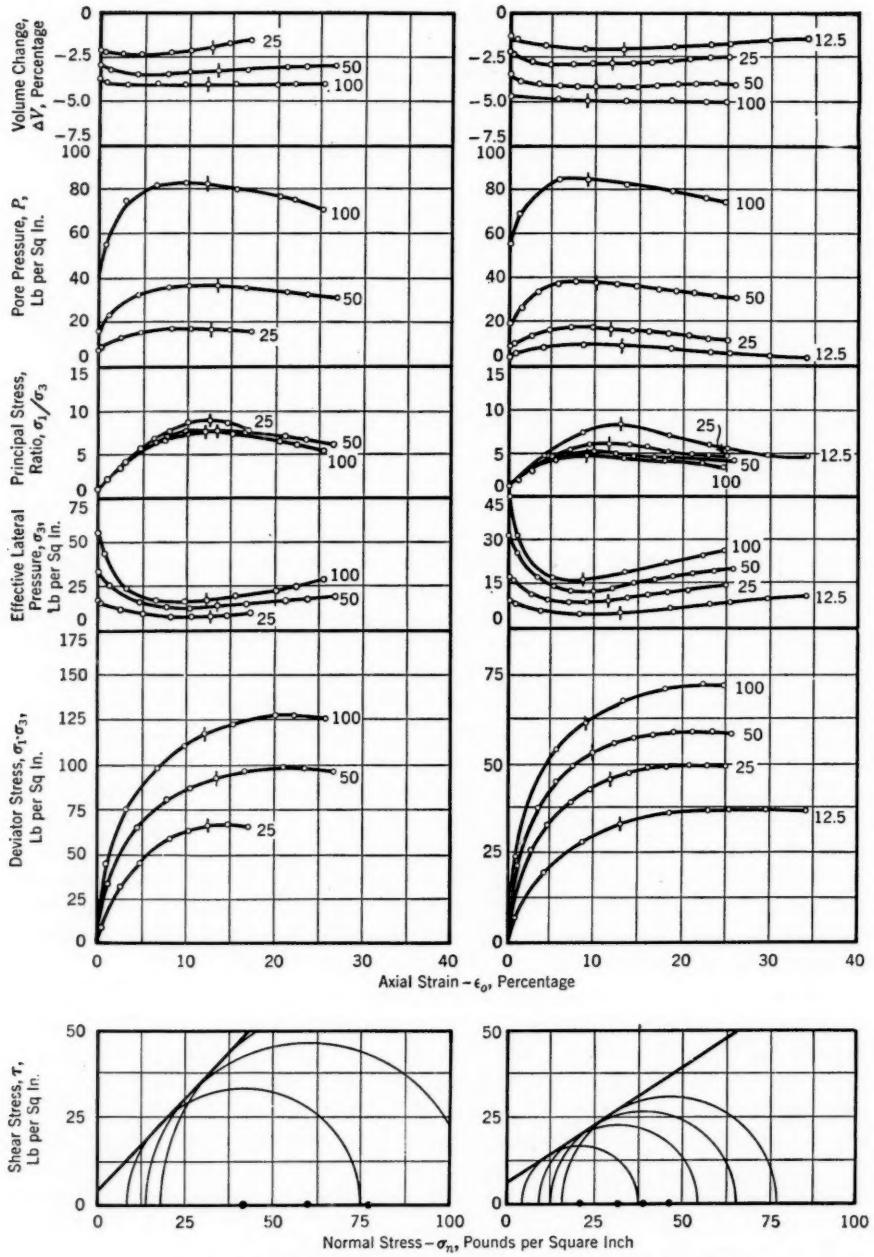
Since the development of modern practices for controlling moisture and compaction in the construction of rolled-earth dams, common field control procedures have been based on the principle that the most desirable condition for the placement of earth materials in embankments is the condition corresponding to the peak point of the moisture density curve. This trend in construction control was based on the theory that materials so placed would provide a fill of minimum permeability, minimum consolidation, minimum water content when saturated, and maximum ultimate stability. For practical reasons, limits for moisture control generally have been based on moisture content corresponding to some percentage of the laboratory maximum density as determined from the density-moisture curve. The concept is extremely broad and illustrative of the tendency to generalize about the performance of a soil mass without giving full consideration to all the factors involved in placing and loading the mass.

The effect of pore pressure on stability assumes increased importance as the placement moisture content is increased, as the fill loads become large, and the permeability of the materials used becomes small. The magnitude of these pore pressures has been actually observed through laboratory and field observations[12] as well as computed on the basis of rational formulas.[13] It was, therefore, considered necessary to reduce construction pore pressures by reducing the placement moisture content as much as possible. It is also well known that a soil mass that is placed at too low a moisture content will consolidate upon saturation. The amount of consolidation is dependent on the placement moisture, the consolidation characteristics of the soil, and the final load conditions.

The USBR believed that the laboratory could furnish an important guide for rolled-fill construction by the establishment of suitable moisture control limits so that neither high pore pressures nor excessive consolidation would result during or after construction. The consolidation test was used primarily for determining these limits. The laboratory tests used and the methods for determining the placement moisture limits have been described in detail elsewhere[14] and will be referred to only briefly in this paper.

Moisture Content Limits.—The lower limit is defined as the lowest placement moisture content at which saturation will have no effect on consolidation under the fill load being considered. The data for determining the limiting moisture contents are presented in terms of dry density, placement moisture, and applied pressures, as shown in Fig. 5. The sample used as a basis for the curves in Fig. 5 was a lean silty clay material with a specific gravity of 2.67 and composed of 24.7% clay size (− 0.005 mm). The four solid lines that were drawn through the unsaturated, consolidated dry densities obtained at each placement moisture condition provide the consolidated dry density versus placement moisture curves for each respective load condition, before consolidation, when drainage is permitted. Similarly, the four dashed lines that were drawn

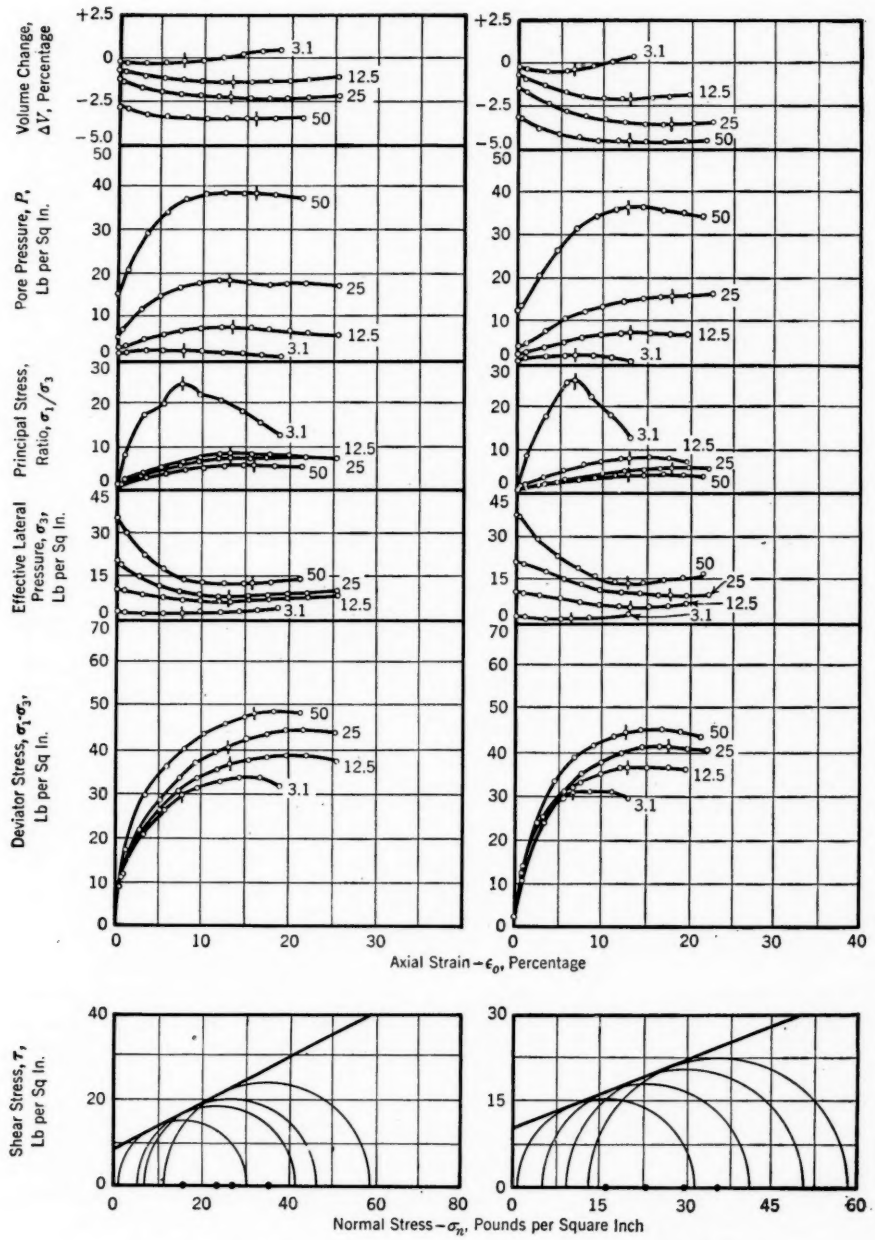
[14] "The Determination of Limits for the Control of Placement Moisture in High Rolled-Earth Dams," by W. G. Holtz, *Proceedings*, American Society for Testing Materials, Vol. 48, 1948, p. 1240.


Volume Change, ΔV, Percentage
Pore Pressure, P, Lb per Sq In.
Principal Stress, Ratio, σ1/σ3
Effective Lateral Pressure, σ3, Lb per Sq In.
Deviator Stress, σ1-σ3, Lb per Sq In.
Axial Strain – εa, Percentage
Shear Stress, τ, Lb per Sq In.
Normal Stress – σn, Pounds per Square Inch

(a) TEST 7–T
SAMPLE 3RX-43
SAND WITH CLAY BINDER

(b) TEST 13–T
SAMPLE 00T-8
CLAY, LEAN SILTY

FIG. 4.—TRIAXIAL SHEAR


Volume Change, ΔV, Percentage
Pore Pressure, P, Lb per Sq In.
Principal Stress, Ratio, σ1/σ3
Effective Lateral Pressure, σ3, Lb per Sq In.
Deviator Stress, σ1-σ3, Lb per Sq In.
Axial Strain −ε0, Percentage
Shear Stress, τ, Lb per Sq In.
Normal Stress −σn, Pounds per Square Inch
3.1
12.5
25
50

(c) TEST 19–T
SAMPLE ATX-3
CLAY MODERATE TO HIGH PLASTICITY

(d) TEST 20–T
SAMPLE ATX-1
CLAY VERY PLASTIC AND VERY COMPRESSIBLE

TESTS ON TYPICAL MATERIALS

through the saturated consolidated dry densities obtained at each placement moisture condition provide the consolidated density versus placement moisture curves after saturation, without change in load. The difference between the solid and dotted lines is the effect of saturation on consolidation for the re-

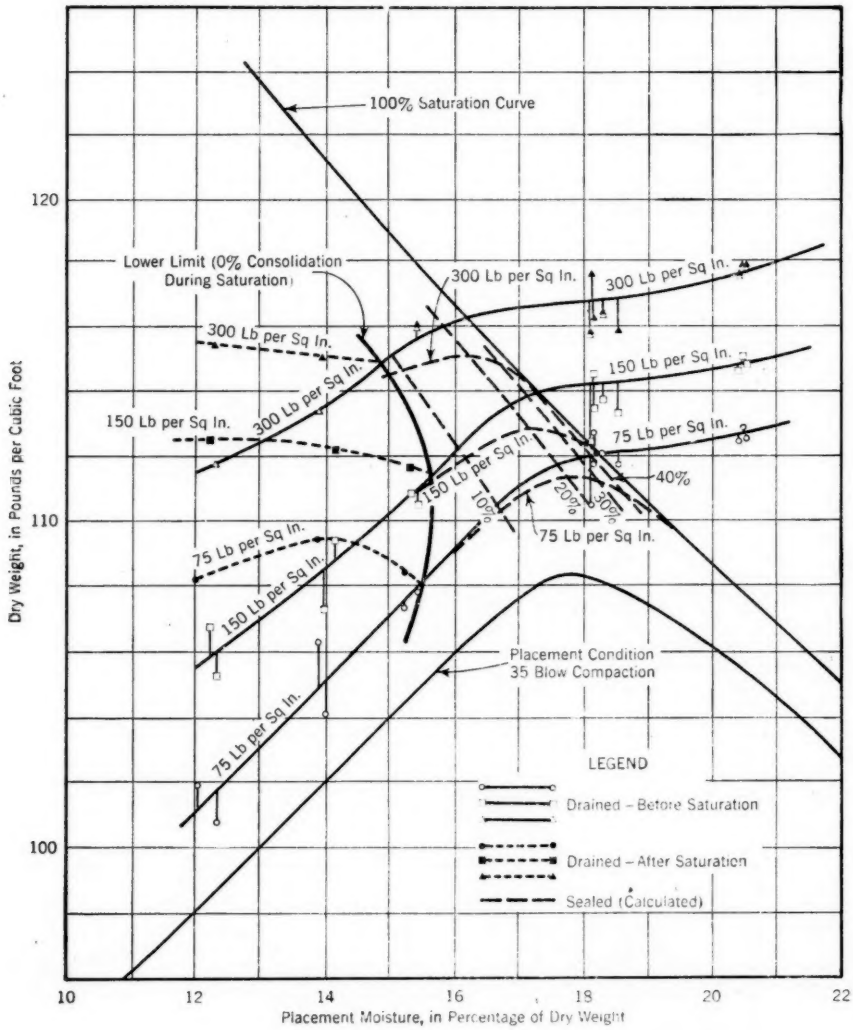


FIG. 5.—LIMITING MOISTURE CURVES

spective loads applied. The intersection of these two lines gives the placement condition at which no additional consolidation will occur upon saturation.

The upper limit is defined as the highest placement moisture content that can be tolerated without producing serious construction pore pressures tending

to reduce the safety factor of the embankment to an unacceptable degree. As practical limitations make it necessary to vary this limit in various parts of the structure, the upper limits are given as heavy dashed lines in Fig. 5 for several pore pressure values in percentage of total load. The upper limit lines are calculated from the results of the one-dimensional consolidation tests, assuming that the specimens are completely sealed and using Eqs. 4 and 5*b*. Calculations were made from the consolidation test data to determine the total applied pressure (pore pressure plus effective pressure) necessary to produce the same consolidation that would occur if the soil were sealed from drainage. This information is shown for the various pressures as the thin dashed lines on Fig. 5 with the applied pressures indicated on the curves. The lines were computed for the total pressures desired by interpolation from the computed data. The heavy dashed lines that cross these thin dashed lines represent the pore pressures that will be obtained for the fill pressures and placement moistures indicated. The positions of these pore pressure lines were obtained by calculating the pore pressures at points along the total pressure lines, using Eq. 5*b*. Pore pressure-consolidation tests that have been used to check the results of these calculations have shown extremely good correlation between the pore pressures computed and those actually obtained from tests.

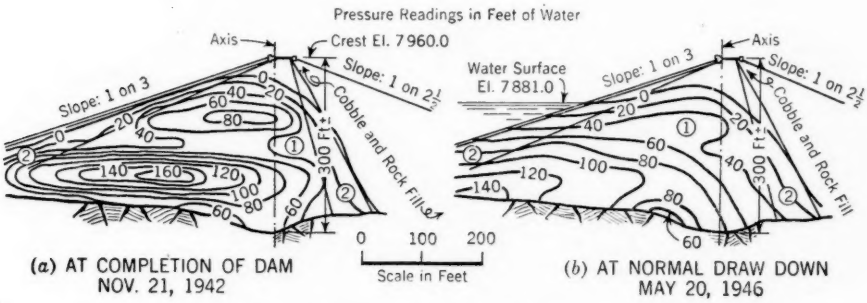


FIG. 6.—PORE PRESSURES IN GREEN MOUNTAIN DAM

Modification of Criteria.—The criteria given were developed for tests on fine-grained impervious soils containing no rock particles. It was recognized that these concepts would have to be modified when considering impervious soils containing greater than about 25% rock. Large-scale consolidation tests were undertaken, therefore, to study these soils from one project in which this type of material was used.[15] These tests, although probably too expensive and lengthy to be standardized for all borrow materials containing a considerable proportion of rock, provided information that can be applied to other soil and rock mixtures. Briefly, it was found that the probability of high pore pressure decreases as the rock content is increased, because the consolidation of the total material is restricted by the increase in rock content. This restriction of consolidation is greater than can be accounted for by the solid rock volume

[15] "The Effect of Rock Content and Placement Density on Consolidation and Related Pore Pressure in Embankment Construction," by H. J. Gibbs, *Proceedings*, American Society for Testing Materials, Vol. 50, 1950, p. 1343.

displacing the compressible fine soil fraction and thus indicates the effect of particle interference. It was further found, that for a given type of grain size distribution curve, 3-in. maximum size material showed less consolidation and the same or less pore pressure than 1-in. maximum size material. This was caused mainly by the corresponding increase in rock content. However, for equal total rock content, the 3-in. maximum size material showed greater consolidation and greater pore pressure than the 1-in. maximum size material. This was attributed to the fact that the 3-in. maximum size provided a better graded material, more adaptable to dense arrangement of particles, than did the 1-in. maximum size.

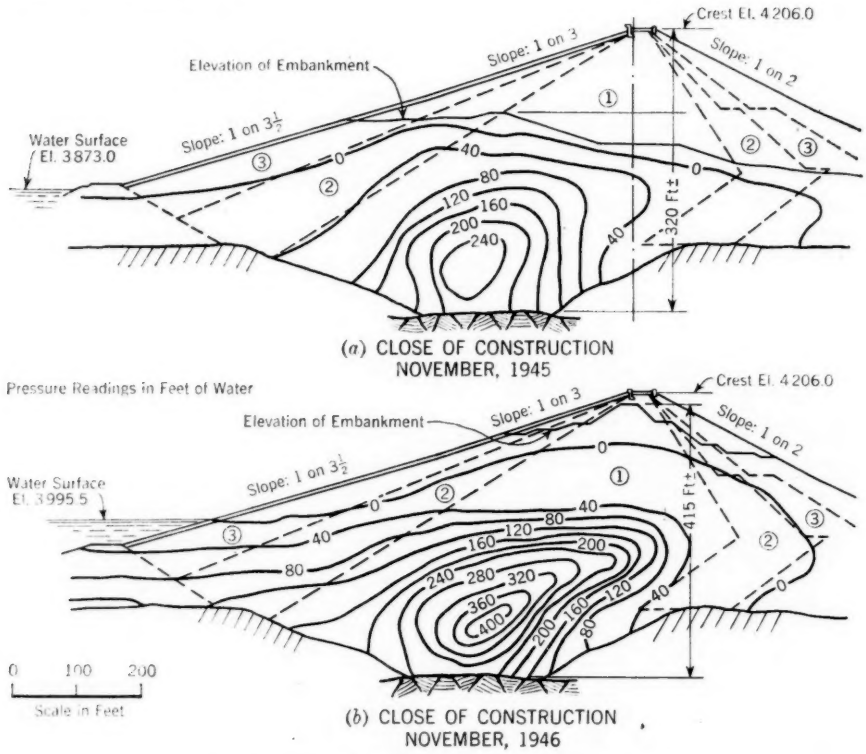


FIG. 7.—PORE PRESSURES IN ANDERSON RANCH DAM

Another fact emphasized by this series of tests was that with the total material placed at 80%, 90%, and 100% of maximum theoretical laboratory density, the less dense material gave much higher consolidation and pore pressure, even though the lower density material was less saturated.

Field Control of Moisture Content.—In 1940, during the construction of Green Mountain Dam, (Blue River, Colo.), a structure more than 300 ft high, it was observed that much higher pore pressures were being developed than had been anticipated on the basis of the knowledge then existent. In some

instances these pressures amounted to 65% of the superimposed fill. The pore pressures, when the fill was completed, are shown on Fig. 6. Fortunately the construction season was limited to the summer months. This allowed some pore pressure dissipation during the winter months. The soil used was a clayey, glacial till that contained about 40% rock fragments larger than No. 4 screen. Abnormally high densities, that in some cases were as high as 150 lb per cu ft, were reported, with the average dry density of a great many tests being 136 lb per cu ft. Placement moisture approximated the laboratory optimum.

As a result of this experience, laboratory studies were initiated to find a means for alleviating these high pressures. The limiting moisture concept was the result. Limited tests indicated that, if field compaction using the rollers as specified by the USBR were performed at a moisture content about 2% dry of the laboratory optimum, a very material reduction in pore pressure would be possible. Immediately following World War II a number of dams were started, but, because of the limited time available, it was impossible to complete limiting moisture tests, and therefore, the above general criterion was adopted for control.

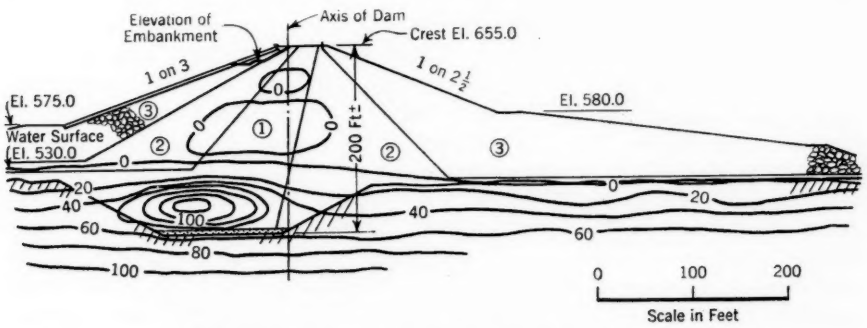


FIG. 8.—PORE PRESSURES IN DAVIS DAM, APRIL 21, 1949

The efforts were only partially successful, not because of any defect in the concept but because it was difficult to overcome the entrenched policy of requiring compaction at laboratory optimum moisture and the recommendation by many authorities that compaction at conditions moister than optimum should be used. In the construction of Anderson Ranch Dam near Mountain Home, Idaho, moisture control approximated laboratory optimum in the lower, more critical areas and the pore pressures developed in the central part of the dam were 80% to 100% of the values predicted for such conditions. The upper part, in which drier placement conditions were attained, shows very little pore pressure. The pore pressure condition at two stages of construction is shown on Fig. 7. In the Horsetooth Reservoir dams near Fort Collins, Colo., and at Jackson Gulch Dam near Moncos, Colo., it was necessary to use soils that would have developed very high pore pressures if compacted at laboratory optimum moisture. Limiting moisture tests were used for control,

and the resultant pore pressures are insignificant. In the cutoff trench at Davis Dam near Kingman, Arizona, materials were used that had not been analyzed for pore pressure. Tests made prior to World War II and before limiting moisture control was inaugurated indicated that it would be difficult to achieve a high impermeability and that pore pressure probably would not be a problem. The first placements were made using material with a water content greater than optimum. By the time the first pore pressure measurements could be made, there was about 50 ft of fill over the pressure cells located in this material. As additional fill was placed, there was a corresponding increase in pore pressure, showing that the pore fluid carried all the additional load of the dam. Corrective measures had been instituted by this time, and the pore pressures in the remainder of the fill are small as can be seen in Fig. 8. This figure also shows what can be done if limiting moisture control is followed. It has been observed, after a material is compressed to the point where the contained moisture completely saturates the material, that any additional load is carried entirely by the pore water. In some soils this condition has been developed with very small loads (less than 10 lb per sq in.), and in many soils it is attained at between 100 and 200 lb per sq in. when optimum moisture is used for compaction.

Summary

The most significant conclusion that can be reached in correlating laboratory data and field performance is not the further refinement of laboratory tests or field measurements but the fact that good correlation can be obtained only if all the factors affecting the field performance are properly accounted for. For instance, the consolidation and pore pressure development of an embankment cannot be properly analyzed unless the placement conditions, the effect of rock particles, and effective pressures are properly considered. With regard to compaction, the present laboratory standards of impact compaction for cohesive soils, or relative density for cohesionless soils, are sufficiently accurate for control purposes. However, it should be realized that moisture and density requirements established by this control must be modified when necessary to conform to the actual density-moisture-compactive effort data obtained from rolling operations in the field at the start of each dam construction operation. From a design standpoint, the results obtained from the standard impact compaction tests are conservative.

One of the greatest needs in studying embankment materials from a laboratory standpoint is for tests of compaction, permeability, consolidation, pore pressure, and shear on soils containing rock particles. This phase of laboratory testing has been neglected because of the length of time and the expensive equipment usually required for these tests. The USBR has tried to fill in a few gaps in this particular phase of testing by development of the large sheepsfoot compaction tests and large-scale permeability and consolidation tests. A research program to study the permeability and consolidation characteristics of several types of soil containing varying amounts of rock particles between the No. 4 and 3-in. size will also produce valuable data. The study of the shear characteristics of such embankment materials has been

particularly neglected, and triaxial equipment has been designed by the USBR for such a study.

On the basis of information available to date (1951) the following tentative conclusions on earth materials containing rock particles have been reached. With a rock content between about zero and $\frac{1}{3}$, the total material behaves very similarly to the soil fraction having particles smaller than the No. 4 size. When the total material contains between about $\frac{1}{3}$ to $\frac{2}{3}$ rock particles, there is theoretically sufficient soil to fill the rock voids, but this may not be true from the practical standpoint of placing the material because as the amount of rock is increased the particle interference is also increased. In this case, increasing rock content may cause an increase in permeability, a decrease in consolidation, and probably an increase in effective shear resistance. Above about $\frac{2}{3}$ rock content in the total material there is insufficient soil to fill the voids by any compaction method, and the material takes on the characteristics of a gravelly or rock-particle material. Permeability in such material will always be relatively high, consolidation relatively low, and frictional resistance should be relatively high.

Any program for correlating laboratory data and field performance should emphasize facilities for collecting measurements for checking the field performance of a structure. Too often structures are built on the basis of laboratory data, and all thought of checking the performance is forgotten unless some unusual condition later takes place. In these latter cases, initial data are usually not available, and a complete study of the situation is impossible. The USBR is attempting to remedy this general situation on those structures under its control by not only installing settlement and pore pressure measuring devices in its earth dams as it has done in the past, but, in addition, establishing bench marks or settlement measuring devices in power plants, pumping plants, and canal structures, with proper initial condition records to enable checking the behavior of structures founded on soil that may have been treated, or untreated, and constructed with or without piles or any combination of these conditions.

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